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5.6 Concrete Slabs

This article now covers only the CCS LRFD superstructure type [BDM 5.6.2]. The transition to the AASHTO LRFD Specifications is complete, and the AASHTO Standard Specifications article [BDM 5.6.1] has been withdrawn.

5.6.1 CCS standard

Withdrawn and archived.

5.6.2 CCS LRFD

5.6.2.1 General [AASHTO-LRFD Section 9, A13]

This series of articles replaces the office document *Design Criteria and Office Practice for Continuous Concrete Slab Bridges* dated 1996.

The design procedures described in this article meet AASHTO LRFD Specifications, Fourth Edition [AASHTO-LRFD Section 9 and A13], with minor modifications. The designer also should review related manual articles for decks [BDM 5.2], railings [BDM 5.8.1], deck drains [BDM 5.8.4], falsework [BDM 7, in process], abutments [BDM 6.5], and piers [BDM 6.6]. The designer should note, however, that at this time not all of the related articles have been updated to the AASHTO LRFD Specifications.

5.6.2.1.1 Policy overview [AASHTO-LRFD 2.5.2.6.3, 4.6.2.1.3, 4.6.2.3, 9.7.1.3]

For relatively short stream and valley crossings with construction conditions that permit use of falsework, the office selects continuous concrete slab (CCS) superstructures.

The office requires that CCS superstructures be designed by the load and resistance factor design (LRFD) method. Early AASHTO LRFD Specifications and associated examples and software were developed with an emphasis on superstructures, and therefore the office decided initially to design CCS superstructures by the LRFD method with HL-93 loading and to meet the AASHTO LRFD Specifications for substructures indirectly by using the AASHTO Standard Specifications with HS-25 loading. As more national LRFD development effort is directed toward substructures the office is making the transition to design of CCS substructures directly by LRFD. A designer for a CCS bridge that does not fit within office standards summarized below shall consult with the supervising Section Leader regarding the design method to be used for the substructure. For a typical CCS bridge with integral abutments and pile bent piers [OBS SS P10A] the question of design method will involve only the foundation piles.

To facilitate the design of typical CCS bridges the office has replaced the office standard sheets with J-series, three-span standards designed for four different roadway widths, 24, 30, 40, and 44 feet (7.315, 9.144, 12,192, and 13.411 m) and four different skews, 0, 15, 30, and 45 degrees. These J24, J30, J40, and J44 standard bridges have superstructures designed according to the AASHTO LRFD Specifications with HL-93 loading but piles designed according to the AASHTO Standard Specifications with HS-25 loading. Spans, lengths, and depths of the standards are summarized in Table 5.6.2.1.1. Ratios between interior and end spans are approximately 1.3 for efficiency. The standard bridges have integral abutments and pile bent piers with a choice of treated timber piles or steel H-piles.

Table 5.6.2.1.1. Lengths, spans, and depths for J24, J30, J40, and J44 three-span continuous concrete slab bridges

Length (1) feet (m)	End Span (2) Feet (m)	Interior Span (3) feet (m)	Depth inches (mm)
70 (21.336)	21.00 (6.401)	28.00 (8.534)	14.50 (368)
80 (24.384)	24.50 (7.468)	31.00 (9.449)	15.25 (387)
90 (27.432)	27.50 (8.382)	35.00 (10.668)	16.25 (413)
100 (30.480)	30.50 (9.296)	39.00 (11.887)	17.50 (445)
110 (33.528)	33.50 (10.211)	43.00 (13.106)	18.50 (470)
120 (36.576)	36.50 (11.125)	47.00 (14.326)	20.00 (508)
130 (39.624)	39.50 (12.040)	51.00 (15.545)	21.25 (540)
140 (42.672)	42.50 (12.954)	55.00 (16.764)	22.50 (572)
150 (45.720)	45.50 (13.868)	59.00 (17.983)	24.00 (610)

Table notes:

(1) Length is measured from centerline of abutment to centerline of abutment.
(2) End span is measured from centerline of abutment to centerline of pier.
(3) Interior span is measured from centerline of pier to centerline of pier.

The standards require revision if loads are increased, if span lengths are varied from those on the plan sheets, or if the number of spans is changed. The standard three-span bridges cannot safely be lengthened with additional interior spans unless the entire superstructure design is re-evaluated and modified.

The depths in the table are 5% to 13% less than the minimum depths required by the AASHTO LRFD Specifications to avoid deflection checks [AASHTO-LRFD 2.5.2.6.3]. The depths rely on the fact that previous standard three-span bridges to 130 feet (39.624 m) long have performed well with respect to deflection. For nonstandard bridges that require redesign due to span arrangements, however, the designer either should meet the AASHTO minimum depth or check deflection.

Cross sections in the standards show a crowned slab of relatively constant thickness formed on the bottom with a double slope. For the builder's use during construction, the office requires the designer to provide top of deck elevations at several longitudinal lines at constant 8- to 10-foot (2.400- to 3.000-meter) intervals on each span.

For typical CCS superstructures, live loads may be distributed by means of the equivalent strip rules [AASHTO-LRFD 4.6.2.1.3].

The office limits skew of continuous concrete slab bridges to 45 degrees. For all skews of 45 degrees or less the primary longitudinal slab reinforcement is designed without regard for skew, neglecting the permissible reduction factor in the AASHTO LRFD Specifications [AASHTO-LRFD 4.6.2.3]. The reinforcement is placed parallel with the longitudinal centerline of the bridge. For skews of 15 degrees or less the transverse reinforcement is placed at the skew angle. For skews greater than 15 degrees the transverse reinforcement is placed perpendicular to the longitudinal centerline of the bridge. The office does not orient longitudinal reinforcement perpendicular to supports for skews greater than 25 degrees [AASHTO-LRFD 9.7.1.3].

Except for unusual design conditions, the office supports continuous concrete slab superstructures on integral abutments and pile bent piers. For typical bridges the office prefers a monolithic cap on a pile bent but, in situations where the monolithic cap would cause excessive stresses or construction difficulties, the designer may use a non-monolithic cap. Plans for the standard three-span CCS bridges include both monolithic and non-monolithic options.

For the relatively shallow slab superstructure, thermal expansion or contraction causes eccentric pressure forces from the fill behind abutments and from the berm in front of abutments. The pressure forces conservatively are assumed to be from full passive pressure. The forces cause end moments that shift reinforcing cutoff points and lengthen the regions of negative moment in end spans but should not reduce positive moment reinforcing. Nonstandard CCS designs need to include the end moments in strength and service limit state checks.

5.6.2.1.2 Design information

The Soils Design Section usually does not provide the specific soil properties necessary for computations of passive pressure for individual bridges. Considering typical site conditions and maintenance procedures, the office conservatively assumes that both the fill behind the abutment and the berm in front of the abutment are cohesive soils. The Soils Design Section recommends the properties in Table 5.6.2.1.2 for typical cohesive soils.

Table 5.6.2.1.2. Recommended soil properties for typical CCS fill and berm soils

Soil Property	**Recommended Design Value**
w, unit weight of soil	0.130 kcf (2080 kg/m^3)
ϕ, angle of internal friction	11.31 degrees (1:5 slope)
c, cohesion	0.600 ksf (28.73 kPa)

If the soils information for a CCS bridge indicates that the fill behind abutments or the soil for the berm will have larger angles of internal friction or cohesion than the recommendations, the designer shall use the properties specific to the bridge site.

5.6.2.1.3 Definitions

Section Leader is the supervisor of the Office of Bridges and Structures preliminary bridge section, final design section, or consultant coordination section.

5.6.2.1.4 Abbreviations and notation [AASHTO-LRFD 3.3.2, 3.10.4.2]

c, cohesion, ksf (kPa)
CCS, continuous concrete slab
DC, dead load of deck, sidewalk, railings, and nonstructural attachments other than utilities [AASHTO-LRFD 3.3.2]
DW, dead load of future wearing surface and any utilities attached directly to the deck [AASHTO-LRFD 3.3.2]
h, depth below top of slab or top of berm, feet (mm)
HPC, high performance concrete
IDC, improved durability concrete
$\mathbf{K_p}$, passive pressure coefficient = $\tan^2 (45 + \phi/2)$
LRFD, load and resistance factor design
$\mathbf{S_{D1}}$**,** horizontal response spectral acceleration coefficient at 1.0-sec. period modified by long-period site factor [AASHTO-LRFD 3.10.4.2]
n, modular ratio
$\mathbf{p_p}$, passive pressure (Rankine Theory), ksf (kPa)
s, shear resistance, ksf (kPa)
w, unit weight of soil, kcf (kg/m^3)
σ, vertical pressure at the horizontal shear plane, ksf (kPa)
ϕ, angle of internal friction

5.6.2.1.5 References

Mahmoudzadeh, M., R.E. Davis, and F.M. Semans. *Modification of Slab Design Standards for Effects of Skew.* Sacramento: California Department of Transportation, 1984.

Office of Construction. *Construction Manual.* Ames: Office of Construction, Iowa Department of Transportation, 2006. (Available on the Internet at: http://www.iowadot.gov/erl/current/CM/Navigation/nav.pdf)

Schneider, E.F. and S.B. Bhide. *LRFD Design of Cast-in-Place Concrete Bridges.* Skokie: Portland Cement Association, 2006.

5.6.2.2 Loads [AASHTO-LRFD 4.6.2.1.4b, 4.6.2.3]

The continuous concrete slabs in the standards are designed as two longitudinal strips: an interior strip and an edge beam. For design the office sets the interior strip 18 inches (450 mm) wide so that it fits a three-bar repetitive pattern for bars spaced at 6 inches (150 mm). The interior strip is derived from an equivalent strip used to distribute live load [AASHTO-LRFD 4.6.2.3].

The edge beam is below the railing and the outer portion of the deck. The edge beam must be designed for a wheel line, and the beam width is governed by rules in the AASHTO LRFD Specifications, which will result in a width much larger than used by the office in the past [AASHTO-LRFD 4.6.2.1.4b]. Usually the width will default to 72 inches (1.829 m). The office requires the edge beam even if it is composite with a concrete barrier rail.

Loads are applied differently to the interior strip and the edge beam, as described in the following articles.

5.6.2.2.1 Dead [AASHTO-LRFD 3.3.2]

For design of typical CCS superstructures the office does not make a distinction between dead load applied before or after curing of the slab. Because the slab is not loaded until the falsework is removed after curing of the slab, all dead load is applied to the same continuous superstructure. All parts of the superstructure are supported by the slab, but usually none are considered to act compositely with the slab. Because of differences in load factors, however, the designer does need to make a distinction between DC, dead load of structural components and nonstructural attachments, and DW, dead load of wearing surfaces and utilities [AASHTO-LRFD 3.3.2].

The interior strip shall be designed to carry the following three dead loads: the weight of the slab strip, the future wearing surface of 0.020 ksf (960 Pa) on the strip, and the strip's share of the weight of the barrier rails distributed over the entire superstructure width.

The edge beam shall be designed to carry the following three dead loads: the weight of the edge beam, the future wearing surface of 0.020 ksf (960 Pa) on the portion of roadway that is part of the beam, and 50% of the barrier rail weight directly above the beam, which generally will be a conservative load distribution. The 50% distribution is discussed in the commentary for this article.

Dead loads for additional superstructure components, such as sidewalks, medians, and utilities, shall be added to the interior strip and edge beam when applicable.

5.6.2.2.2 Live [AASHTO-LRFD 3.6.1.1.2, 3.6.1.2, 3.6.1.3.3, 3.6.1.6, C3.6.1.2.1, C3.6.1.3.1]

Unless special requirements govern the design, vehicular live load (LL) for CCS superstructures shall be HL-93 [AASHTO-LRFD 3.6.1.2]. It shall be applied as discussed in the AASHTO LRFD Specifications [AASHTO-LRFD 3.6.1.3.3].

In unusual situations identified by the Chief Structural Engineer, where a bridge will experience a high percentage of truck traffic, accumulation of trucks due to traffic flow control, or special industrial loads, the designer will need to consider additional loading as discussed in the AASHTO LRFD Specifications commentary [AASHTO-LRFD C3.6.1.2.1, C3.6.1.3.1]. In these situations the designer shall consult with the Chief Structural Engineer.

Sidewalk and other live loads shall be added to the superstructure live load when applicable. A pedestrian load (PL) of 0.075 ksf (3.60 kPa) shall be applied to all sidewalks [AASHTO-LRFD 3.6.1.6] and considered simultaneously with the vehicular live loads as specified for multiple presence of live load [AASHTO-LRFD 3.6.1.1.2].

The interior strip, as well as the edge beam, shall be designed for live load with main reinforcement parallel with traffic regardless of skew, but skew shall not exceed 45 degrees.

5.6.2.2.3 Dynamic load allowance [AASHTO-LRFD 3.6.2.1]

The dynamic load allowance (IM) shall be taken from the AASHTO LRFD Specifications [AASHTO-LRFD 3.6.2.1]. It shall be applied to truck and tandem live loads on the interior strip and on the edge beam.

5.6.2.2.4 Railing [AASHTO-LRFD A13]

Railing loads for design of the transverse reinforcing near the slab edge shall be in accordance with the AASHTO LRFD Specifications [AASHTO-LRFD A13]. Except in the case of unusual design criteria, design forces for traffic railings shall be taken at Test Level Four (TL-4). For interstate bridges, however, traffic volume and mix, unfavorable site conditions, and other factors may require Test Level Five or Six (TL-5 or TL-6). The designer shall verify exceptions to Test Level Four (TL-4) with the supervising Section Leader.

The transverse slab reinforcing near the railings is designed for the standard TL-4 or TL-5, F-shape barrier rail and open rail shown on the J-series plans. If TL-6 barrier rails or other railings are used, the designer shall check and, if necessary, redesign the transverse slab reinforcing. Yield line values required for TL-4 and TL-5, F-shape barrier rails are given in the decks section of this manual [BDM 5.2.2.4].

The railing load applied to the slab is intended to make the slab stronger than the railing so that an overload will cause collapse of the railing but not of the slab. Therefore, the designer should be cautious about strengthening a barrier rail in order to avoid requiring additional transverse reinforcement for the slab.

5.6.2.2.5 Earth pressure

As the CCS superstructure expands or contracts with temperature change, the superstructure will be restrained by earth pressure (EH). For the typical superstructure with integral abutments, the movement conservatively is assumed to be large enough to develop passive earth pressure at the service limit state regardless of bridge length.

As the superstructure expands, passive pressure will develop in the fill behind the abutment. Although the fill behind the abutment initially is granular, the region behind the abutment sometimes is mud-pumped to fill voids as part of approach slab maintenance operations. The mud-pumped region then is likely to push against cohesive soil beyond the granular fill region. Therefore the office conservatively assumes that the fill behind the abutment is cohesive soil.

As the superstructure contracts, passive pressure will develop in the berm, which also is assumed to be cohesive soil. The designer should check the berm at the service limit state to ensure that the force developed on the horizontal shear surface is larger than the force from the passive pressure that will develop in front of the abutment. The service limit state passive pressures and shear surface are illustrated in Figure 5.6.2.2.5.

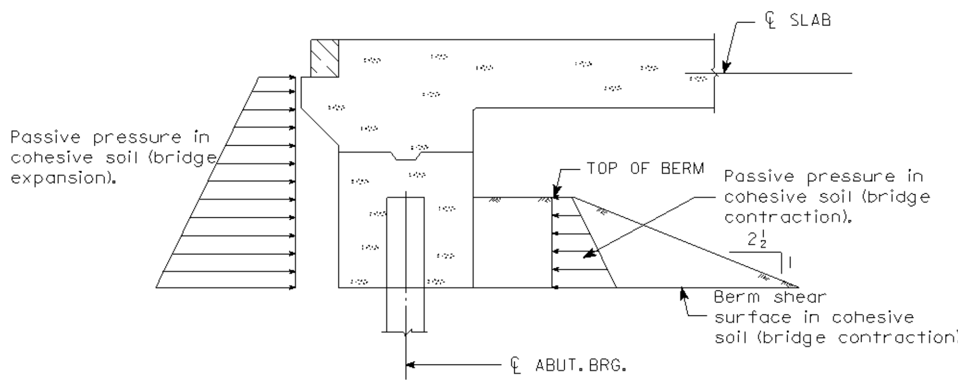

Figure 5.6.2.2.5. Service limit state passive pressures and shear surface at a CCS integral abutment

For computing the service limit state passive pressures and the shear resistance of the berm the designer shall use the following formulas with the soil properties recommended for the bridge site and abutment details [BDM 5.6.2.1.2].

$$p_p = whK_p + 2c\sqrt{K_p}$$

Where:

p_p = passive pressure (Rankine Theory, service limit state), ksf (kPa)
w = unit weight of soil, kcf (kg/m^3)
h = depth below top of slab or top of berm, feet (mm)
K_p = passive pressure coefficient = tan^2 (45 + ϕ/2)
ϕ = angle of internal friction
c = cohesion, ksf (kPa)

$$s = c + \sigma \tan\phi$$

Where:

s = shear resistance (service limit state), ksf (kPa)
c = cohesion, ksf (kPa)
σ = vertical pressure at the horizontal shear plane, ksf (kPa)
ϕ = angle of internal friction

5.6.2.2.6 Earthquake [AASHTO- LRFD 3.10.3.1, 3.10.4.2, 3.10.6, 4.7.4.1]

Based on the acceleration coefficient S_{D1}, all of Iowa with Site Class A through E shall be classified as Seismic Zone 1 [AASHTO-LRFD 3.10.3.1, 3.10.4.2, 3.10.6]. Thus for typical bridges no seismic loading (EQ) or analysis is required [AASHTO-LRFD 4.7.4.1]. However, for unusual projects such as bridge sites determined to be Site Class F and for Missouri River and Mississippi River bridges the designer shall determine the seismic zone and perform seismic analysis as required by the AASHTO LRFD Specifications.

5.6.2.2.7 Construction

For most bridge projects it is assumed that construction can take place without cranes, construction vehicles, construction equipment, and concentrated quantities of construction materials on the bridge. If, however, the contractor does need to place loads on the bridge larger than those permitted by the Standard Specifications [IDOT SS 1105.12, D], the contractor will be required to submit structural analysis by an Iowa-licensed engineer for approval. Thus the bridge designer may be required to review construction loading after letting of the bridge contract.

5.6.2.3 Load application to superstructure

5.6.2.3.1 Load modifier [AASHTO-LRFD 1.3.2, 3.4.1]

Load factors shall be adjusted by the load modifier, which accounts for ductility, redundancy, and operational importance [AASHTO-LRFD 1.3.2, 3.4.1]. For typical CCS bridges the load modifier shall be taken as 1.0.

5.6.2.3.2 Limit states [AASHTO-LRFD 3.4.1, 3.4.2]

For the typical CCS bridge superstructure, the designer shall consider the following limit states [AASHTO-LRFD 3.4.1].

- Strength I, superstructure with vehicles but without wind
- Extreme Event II, collision by vehicle (railing)
- Service I, superstructure with vehicles and 55 mph (89 kph) wind
- Fatigue

Although other limit states are unlikely to control for typical bridges, the designer should be alert to special conditions that require consideration of additional limit states.

During construction the superstructure will be supported on the substructure as well as on falsework, which will carry most of the construction loads. The falsework designer shall consider all appropriate construction loads and load combinations in design of the falsework.

Load combinations and load factors for limit states are given in the AASHTO LRFD Specifications [AASHTO-LRFD 3.4.1, 3.4.2].

5.6.2.4 Continuous slabs

5.6.2.4.1 Analysis and design

The office requires the LRFD method for design of CCS superstructures.

5.6.2.4.1.1 Analysis assumptions

For CCS bridges supported by typical integral abutments and pile bents, the designer may consider the interior strip and edge beam to be continuous beams with downward extensions for lateral soil loads at abutments and with simple supports at abutments and pile bents. However, if piles supporting abutments are less than 15 feet (4.570 m) long, the designer shall evaluate the stiffness of the piles with respect to the stiffness of the slab superstructure. It may be necessary to analyze the slab superstructure along with the piles as a continuous frame.

5.6.2.4.1.2 Materials

Unless otherwise specified, concrete for the slab shall be structural concrete, Class C, as defined in the standard specifications [IDOT SS 2403]. Class C concrete shall be assumed to have a 28-day strength of 4.0 ksi (28 MPa).

If requested by the District and approved by the Assistant Bridge Engineer, high performance concrete (HPC) will be specified for the slab and other bridge components, and the designer shall consult the developmental specification prepared by the Office of Materials for the 28-day concrete strength [IDOT DS-09012, Developmental Specification for High Performance Concrete for Structures].

If requested by District 3 and approved by the Assistant Bridge Engineer, improved durability concrete (IDC) will be specified for the slab and related bridge components. The IDC concrete will have the same strength as Class C concrete. For further information the designer shall consult the developmental specification prepared by the Office of Materials [IDOT DS-09011, Developmental Specification for Improved Durability Concrete for Bridge Decks].

Unless otherwise specified, reinforcement shall be ASTM A 615/A 615M or ASTM A 996/A 996M, Grade 60 (Grade 420).

5.6.2.4.1.3 Design resistance [AASHTO-LRFD 5.5.4.2.1]

Resistance factors for the slab shall be taken from the AASHTO LRFD Specifications [AASHTO-LRFD 5.5.4.2.1].

5.6.2.4.1.4 Strip properties [AASHTO-LRFD 2.5.2.6.3, 4.6.2.1.4b, 5.7.1]

Depth of the slab for a nonstandard superstructure should be set near the traditional minimum depth determined from span-to-depth ratios [AASHTO-LRFD 2.5.2.6.3]. The designer may use less than the minimum depth with appropriate deflection checks.

For the interior strip and the edge beam the uppermost one-half inch (13 mm) shall be neglected. Neglecting the top surface accounts for longitudinal grooving for texture and pavement markings and for the eventual loss of the roadway built-in wearing surface.

For transforming Grade 60 (Grade 420) tension reinforcement to Class C concrete at the service and fatigue limit states the modular ratio, n, shall be taken as 8 [AASHTO-LRFD 5.7.1]. For transforming Grade 60 (Grade 420) compression reinforcement to Class C concrete, the transformation shall be made using 2n-1. Subtracting one accounts for the concrete displaced by the reinforcement.

For the standard CCS superstructures the longitudinal reinforcing is spaced laterally at 6 inches (150 mm) in three distinct, repetitive lines. Thus it is convenient to set the width of the interior strip at 18 inches (450 mm) for design.

For determining moments, the edge beam width shall be taken as required in the AASHTO LRFD Specifications [AASHTO-LRFD 4.6.2.1.4b].

The railing shall not be included in the section properties for the edge beam.

5.6.2.4.1.5 Moment [AASHTO-LRFD 5.7.3.4 before 2005 Interim]

The designer shall use maximum moment envelopes for the Strength I limit state to select reinforcement and locate initial cutoff points. The envelopes shall be plotted for moments at the tenth points of the span or a closer spacing. The designer may use linear interpolation between envelope points.

Using Service I limit state moment envelopes, the designer shall adjust cutoff points as necessary to provide additional reinforcement for the z-method of crack control in effect before the 2005 Interim [AASHTO-LRFD 5.7.3.4 before 2005 Interim]. For the z-method the designer shall perform the check using computed service limit state stresses and the severe exposure condition.

The moment envelopes shall include the positive and negative effects of end moments caused by passive soil pressures as the bridge expands or contracts. For both the interior strip and the edge beam the passive soil pressures affect the moment diagrams and have significant effects on the envelopes for the end spans, as well as moderate effects on the interior spans.

The designer shall take the design moment at each interior support as the maximum negative moment at the center of the support.

At all sections other than above an interior support the designer shall select reinforcing assuming a singly reinforced cross section. Because an interior support confines the compression region at the bottom of the slab, the section may be designed as a doubly reinforced section.

For the interior strip the designer should place the reinforcing in A, B, and C lines, each with different bar sizes and cut-offs. The designer shall select reinforcing for the edge strip so as to maintain the longitudinal reinforcing pattern for the interior strip from gutter line to gutter line. The remainder of the edge strip reinforcing shall be placed below its railing in D and E lines. The very edge of the slab shall have continuous bars at the corners of the cross section.

5.6.2.4.1.6 Shear [AASHTO-LRFD 4.6.2.3, 5.14.4.1]

CCS superstructures designed by the equivalent strip method [AASHTO-LRFD 4.6.2.3] may be considered satisfactory for shear [AASHTO-LRFD 5.14.4.1].

5.6.2.4.1.7 Camber and deflection [AASHTO-LRFD 2.5.2.6.2, 5.7.3.6.2]

Formwork shall be cambered to account for the total short-term and long-term dead load deflection [AASHTO-LRFD 5.7.3.6.2]. The designer shall include a diagram on the plans indicating the required camber at the quarter points of each span for spans not exceeding 40 feet (12.192 m). For longer spans the designer shall indicate the required camber at the sixth points of the span so that the camber is given at intervals not exceeding 10 feet (3.048 m).

For nonstandard CCS superstructures, the designer shall check deflection for live load plus dynamic load allowance. The deflection shall be limited to 1/800 of each span for bridges designed for vehicles and to 1/1000 of each span for bridges designed for both pedestrians and vehicles [AASHTO-LRFD 2.5.2.6.2].

5.6.2.4.1.8 Fatigue [AASHTO-LRFD 5.5.3.1, 5.3.3.2]

The designer shall check the fatigue limit state as required for longitudinal reinforcement [AASHTO-LRFD 5.5.3.1, 5.3.3.2].

5.6.2.4.1.9 Additional considerations [AASHTO-LRFD 5.10.8, 5.14.4.1, 13.6.2, A13]

Transverse distribution reinforcement shall be placed in the bottom of the slab in accordance with the AASHTO LRFD Specifications [AASHTO-LRFD 5.14.4.1]. The office prefers that the bars be placed at 12 inches (300 mm). See Table 5.6.2.4.2 for direction of all bottom and top transverse reinforcement.

Transverse reinforcement shall be placed in the top of the slab for the greater of the requirements for shrinkage and temperature [AASHTO-LRFD 5.10.8] and for Extreme Event II lateral load on the railing

[AASHTO-LRFD 13.6.2]. The office prefers that shrinkage and temperature bars be placed at 12 inches (300 mm) across the entire slab width, and that for additional capacity short #5 bars (j-bars) be placed below the railings and between the temperature bars.

For typical CCS superstructures Extreme Event II lateral load will control transverse reinforcement near the railings. The lateral load shall be applied at the required height and over the required length of railing. For a concrete barrier rail the critical section for determining the reinforcing in the top of the slab shall be the inside face of the rail. If not given in this manual [BDM 5.2.2.4], the length of the slab section resisting the lateral load shall be as determined from the AASHTO LRFD Specifications [AASHTO-LRFD A13], and the reinforcing shall be extended a development length beyond the section. On standard plans the lateral load reinforcement is provided by 5d and 5j1 bars in the top of the slab.

5.6.2.4.2 Detailing

Except for CCS bridges that require superelevation, the top of the slab shall be crowned in the transverse direction with a central parabolic profile and 2.0% slopes toward each railing. The bottom of the slab shall be pitched along straight lines from the centerline of the bridge so that the slab remains a relatively constant thickness.

So that the contractor may set the proper top of slab elevations, the elevations are to be shown in the longitudinal direction at centerlines of piers, centerlines of abutments, and intervals of 8 to 10 feet (2.400 to 3.000 m) along each span. Top of slab elevations in the transverse direction shall be shown at centerline of the approach roadway, end of parabolic crown each side of centerline, each gutter line, midpoint between end of parabolic crown and gutter line, and longitudinal construction joint, if required for staged construction.

For the CCS superstructure, cover shall be 2 inches (50 mm) unless otherwise noted on plans. For the longitudinal steel, cover shall be 2 ½ inches (65 mm) for top steel and 1 ½ inches (38 mm) for bottom steel. Note that the bottom steel cover is larger than the cover for a concrete deck supported on beams or girders [BDM 5.2.2.4.2].

Minimum practical spacing for main bars is 4½ inches (115 mm). The usual spacing is 6 inches (150 mm). Maximum bar size for slab reinforcing shall be #11 (#36).

When two different bar sizes are spliced together, assume the cut-off point for the larger bar is at the center of the splice.

If the cut-off point of a bottom bar is within 2 feet (600 mm) of the centerline of abutment bearing, extend the bar to the end of the slab. If at all possible, reduce bar size or otherwise alter the reinforcing for the slab so that bars can be developed without hooks.

All reinforcement in CCS superstructures on the primary highway system shall be epoxy coated except spirals, including the following:

- All deck slab reinforcing steel, longitudinal and transverse, top and bottom.
- All abutment reinforcing steel.
- All reinforcing steel in pier caps including cap steel from P10A piles.
- All barrier rail and median barrier reinforcing steel, longitudinal and vertical.
- All light pole base reinforcing steel.
- All wing reinforcing steel.

All reinforcement in the J40 and J44 standard bridges, which typically are used for state highways, is epoxy coated as indicated above. However, the J24 standard bridges are designed with black steel, and the J30 standard bridges have the option of black or epoxy coated steel. Generally the narrower standard bridges are intended for county roads and other locations off the state highway system.

The designer shall provide a permissible longitudinal construction joint at the centerline of the bridge whenever the bridge width exceeds 32 feet (9.600 m) or whenever staged construction is required. Because the office requires falsework to remain in place across the entire bridge width until both sides of the deck are complete there is no need for a closure pour at the joint. Longitudinal reinforcement may be shifted 1 inch (25 mm) to miss the joint. Transverse reinforcement should be spliced near the joint.

CCS bridges shall have permissible transverse construction joints near the dead load inflection points. The joints are to be configured as specified in Table 5.6.2.4.2.

Table 5.6.2.4.2. Transverse reinforcement and construction joints for CCS superstructures

Skew	**Direction of Transverse Reinforcement**	**Configuration of Transverse Joints**	**Keyway, nominal size [(1)]**
15 degrees or less	Parallel with skew	Parallel with skew but perpendicular to longitudinal centerline below barrier rails	Beveled 2 x 6 (50 mm x 150 mm)
More than 15 degrees to 30 degrees	Perpendicular to longitudinal centerline	Parallel with skew but perpendicular to longitudinal centerline below barrier rails	Beveled 2 x 6 (50 mm x 150 mm)
More than 30 degrees	Perpendicular to longitudinal centerline	Stepped; parallel with skew alternate, Figure 5.6.2.4.2	Beveled 2 x 6 (50 mm x 150 mm) along entire joint

Table note:

(1) See CADD Note E443/M443 [BDM 11.5.2].

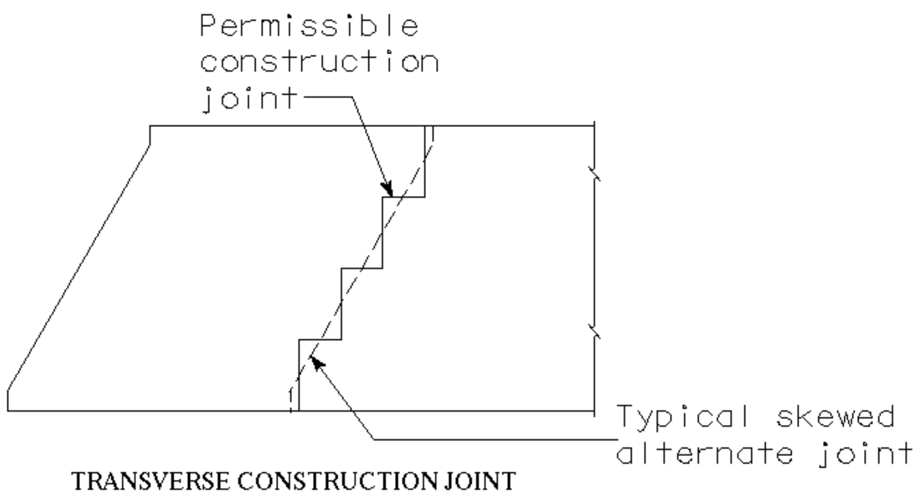

Figure note:

- See also the J44-series, sheet J44-22-06 for parallel joints and sheet J44-24-06 for stepped joints.

Figure 5.6.2.4.2. Stepped transverse construction joint

Because the office requires that falsework remain in place until the entire slab superstructure is placed and cured there usually is no need to require a deck placement sequence. If unusual circumstances require a sequence, the designer shall indicate the sequence on the plans.

The designer shall provide deck drains appropriate for the slab slope, railing type, bridge substructure, and site. For J30, J40, and J44 standard bridges with barrier rail the standard plans [OBS J30-06, J40-06, and J44-06] give recommended deck drain details and locations. These locations may need to be adjusted for project conditions. See the commentary for decks for additional deck drain guidelines [BDM C5.2.4.1.2].